Figure IV-3-39. Ridge and runnel north of St. Joseph, Michigan, November 1993. This example shows that these features can be found on lake shores that do not have regular tides

(j) Gravity-induced downslope transport.

(2) Additional complications are imposed by constantly changing shoreface conditions, as follows:

(a) The relative contributions made by the different transport mechanisms vary over time.

(b) Because of differing regional geological configuration and energy climate, the frequencies of occurrence of the different mechanisms vary with location.

(c) Oscillatory flows normally occur at many frequencies and are superimposed on mean flows and other oscillatory flows of long period.

(3) Middle Atlantic Bight experiments of Wright et al. (1991).

(a) Wright et al. (1991) measured suspended sediment movement, wave heights, and mean current flows at Duck, North Carolina, in 1985 and 1987 and at Sandbridge, Virginia, in 1988 using instrumented tripods. During their study, which included both fair weather and moderate energy conditions, onshore mean flows (interpreted to be related to tides), were dominant over incident waves in generating sediment fluxes. In contrast, during a storm, bottom conditions were strongly dominated by offshore-directed, wind-induced mean flows. Wright et al. attributed this offshore-directed flow to a rise of 0.6 m in mean water level (during this particular storm) and a resultant strong seaward-directed downwelling flow.

(b) Wright et al. (1991) examined the mechanisms responsible for onshore and offshore sediment fluxes across the shoreface. They related two factors explicitly to incoming incident waves:

- Sediment diffusion arising from gradients in wave energy dissipation.

- Sediment advection caused by wave orbital asymmetries.

They found that four other processes may also play important roles in moving sediment:

- Interactions between groupy incident waves and forced long waves.

- Wind-induced upwelling and downwelling currents.

- Wave-current interactions.

- Turbidity currents.

Overall, Wright et al. found that incoming incident waves were of primary importance in bed agitation, while tide- and wind-induced currents were of primary importance in moving sediment. The incoming wave orbital energy was responsible for mobilizing the sand, but the unidirectional currents determined where the sand was going. Surprisingly, cross-shore sediment fluxes generated by mean flows were dominant or equal to sediment fluxes generated by incident waves in all cases and at all times.

(c) Based on the field measurements, Wright et al. (1991) concluded that "near-bottom mean flows play primary roles in transporting sand across isobaths on the upper shoreface" (p. 49). It is possible that this dominance of mean flows is a feature that distinguished the Middle Atlantic Bight from other shorefaces. The oscillatory (wave) constituents may be proportionately much more important along coasts subject to persistent, high-energy swell, such as the U.S. west coast. Wright et al. also concluded that the directions, rates, and causes of cross-shore sediment flux varied temporally in ways that were only partly predictable with present theory.

f. Sea level change and the Bruun rule.

(1) General coastal response to changing sea level.[1] Many barrier islands around the United States have accreted vertically during the Holocene rise in global sea level, suggesting that in these areas the supply of sediment was sufficient to allow the beaches to keep pace with the rise of the sea. It is not clear how beaches respond to short-term variations in sea level. Examples of shorter processes include multi-year changes in Great Lakes water levels and multi-month sea level rises associated with the El Niño-Southern Oscillation in the Pacific.

[1] Part IV-1 reviewed sea level change and outlined some of the associated coastal effects and management issues. Table IV-1-7 outlined how shoreline advance or retreat at any particular location is a balance between sediment supply and the rate of sea level change. In this section, sea level change is meant in a general sense to be caused by a combination of factors, including eustatic (global) changes and local effects due to vertical movements of the coastal land.

(2) Storm response.

(a) Based on his pioneering research of southern California beaches in the 1940's, Shepard (1950) developed the classic model that there is an onshore-offshore exchange of sediment over winter-summer cycles. Studies since then have shown that this model applies mostly to beaches on swell-dominated coasts where the wave climate changes seasonally (particularly Pacific Ocean coasts) (Carter 1988). Many beaches do *not* show an obvious seasonal cycle. Instead, they erode during storms throughout the year and rebuild during subsequent fair weather periods. Some coasts, like New Jersey, have a seasonal signature, but storms cause such great perturbations that it can take repetitive surveys over many years to extract the seasonal signature.

(b) In some locations, such as the Gulf Coast, infrequent and irregular hurricanes may be the most important dynamic events affecting beaches. Following one of these storms, beach and dune rebuilding may take years (Figure IV-2-10 shows a portion of the Florida/Alabama shore that was damaged by Hurricane Frederic in 1979 and is slowly recovering). Recently, the popular belief that hurricanes are the most important morphodynamic events causing Gulf Coast beach erosion is being reevaluated with the benefit of new field data. Scientists have learned that, cumulatively, winter cold fronts produce significant annual barrier island retreat. Dingler, Reiss, and Plant (1993) monitored Louisiana's Isles Dernieres and found that Hurricane Gilbert (September 1988) produced substantial beach retreat initially, but it actually reduced the average erosion rate by modifying the slope of the shoreface from that produced by cold-front-generated storms. The different responses were related to the scale of the storms. Cold fronts, which individually were small storms, eroded the entire beach to the same degree. Most sand and mud was deposited offshore and only a small percentage of eroded sand was deposited on the backshore because the fronts usually did not raise the sea enough to cause overtopping. Hurricane Gilbert, in contrast, raised sea level substantially such that the primary erosion occurred on the upper beach, and much of the sand was deposited behind the island via overwash processes. Over a 5-year period, the overall effect of this hurricane on the Isles Dernieres was to retard the retreat rate of the island by about 50 percent over that produced by cold fronts alone.

(3) Bruun Rule beach response model.

(a) One of the best-known shoreface response models was proposed by Bruun in 1962 (rederived in Bruun (1988)). Bruun's concept was that beaches adjust to the dominant wave conditions at the site. He reasoned that beaches had to respond in some manner because clearly they had adjusted and evolved historically as sea level had changed. Beaches had not disappeared, they had moved. How was this translation accomplished? Earlier studies of summer/winter beach morphology provided clues that beaches responded even to seasonal changes in wave climate. The basic assumption behind Bruun's model is that with a rise in sea level, the equilibrium profile of the beach and the shallow offshore moves upward and landward. Bruun made several assumptions in his two-dimensional analysis:

- The upper beach erodes because of a landward translation of the profile.
- Sediment eroded from the upper beach is deposited immediately offshore; the eroded and deposited volumes are equal (i.e., longshore transport is not a factor).
- The rise in the seafloor offshore is equal to the rise in sea level. Thus, offshore, the water depth stays constant.

(b) The Bruun Rule can be expressed as (Figure IV-3-40a):

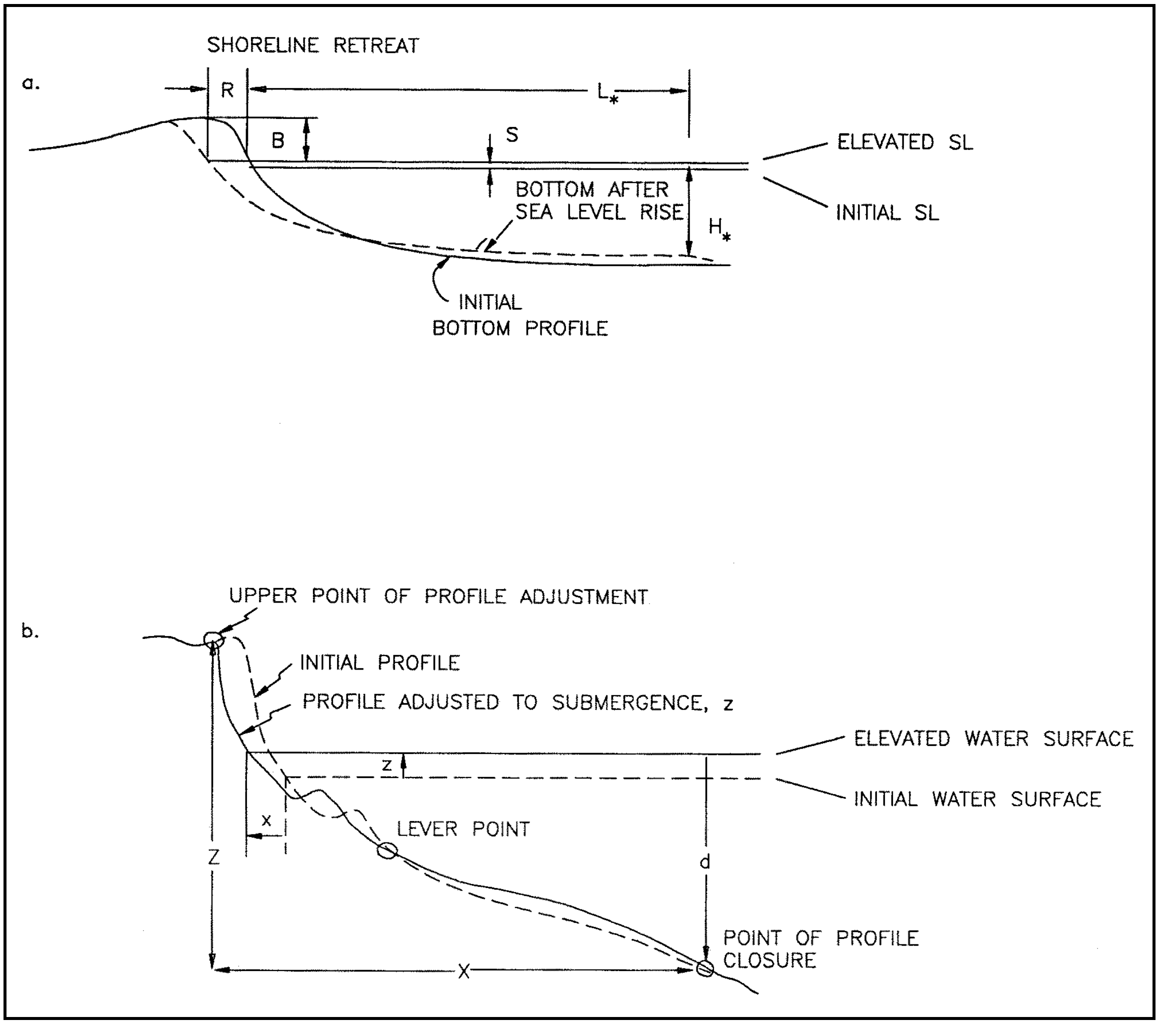

Figure IV-3-40. (a) Shoreline response to rising sea level (SL) depicted by the Bruun Rule. (b) Simplified nomenclature used by Hands (1983). The sandbar shows that the model is valid for complicated profile shapes

$$R = \frac{L_*}{B + H_*} S \qquad \text{(IV-3-5)}$$

where

R = shoreline retreat

S = increase in sea level

L_* = cross-shore distance to the water depth H_*

B = berm height of the eroded area

Hands (1983) restated the Bruun Rule in simplified form:

$$x = \frac{zX}{Z} \tag{IV-3-6}$$

where z is the change in water level. The ultimate retreat of the profile x can be calculated from the dimensions of the responding profile, X and Z, as shown in Figure IV-3-40b. Other expressions for the Bruun Rule are presented in Part III-3-h.

(c) Despite the continued interest in Bruun's concept, there has been only limited use of this method for predictive purposes. Hands (1983) listed several possible reasons for the reluctance to apply this approach:

- Skepticism as to the adequacy of an equilibrium model for explaining short-term dynamic changes.

- Difficulties in measuring sediment lost from the active zone (alongshore, offshore to deep water, and onshore via overwash).

- Problems in establishing a realistic closure depth below which water level changes have no measurable effect on the elevation or slope of the seafloor.

- The perplexity caused by a discontinuity in the profile at the closure depth which appeared in the original and in most subsequent diagrams illustrating the concept.

An additional, and unavoidable, limitation of this sediment budget approach is that it does not address the question of *when* the predicted shore response will occur (Hands 1983). It merely reveals the horizontal distance the shoreline must *ultimately* move to reestablish the equilibrium profile at its new elevation under the assumptions stated in Bruun's Rule.

(d) Hands (1983) demonstrated the geometric validity of the Bruun Rule in a series of figures which show the translation of the profile upward and landward (the figures are two-dimensional; volumes must be based on unit lengths of the shoreline):

- Figure IV-3-41a: The equilibrium profile at the initial water level.

- Figure IV-3-41b: The first translation moves the active profile up an amount z and reestablishes equilibrium depths below the now elevated water level. Hands defines the *active profile* as the zone between the closure depth and the upper point of profile adjustment. The volume of sediment required to maintain the equilibrium water depth is proportional to X (width of the active zone) times z (change in water level).

- Figure IV-3-41c: The required volume of sediment is provided by the second translation, which is a recession (horizontal movement) of the profile by an amount x. The amount of sediment is proportional to x times Z, where Z is the vertical extent of the active profile from the closure depth to the average elevation of the highest erosion on the backshore.

- Figure IV-3-41d: Equating the volume required by the vertical translation and the volume provided by the horizontal translation yields Equation 3-6. In reality, both translations occur simultaneously, causing the closure point to migrate upslope as the water level rises.

(e) One of the strengths of the Bruun concept is that the equations are valid regardless of the shape of the profile, for example, if bars are present (Figure IV-3-40b). It is important that an offshore distance and depth of closure be chosen that incorporate the entire zone where active sediment transport occurs. Thereby,

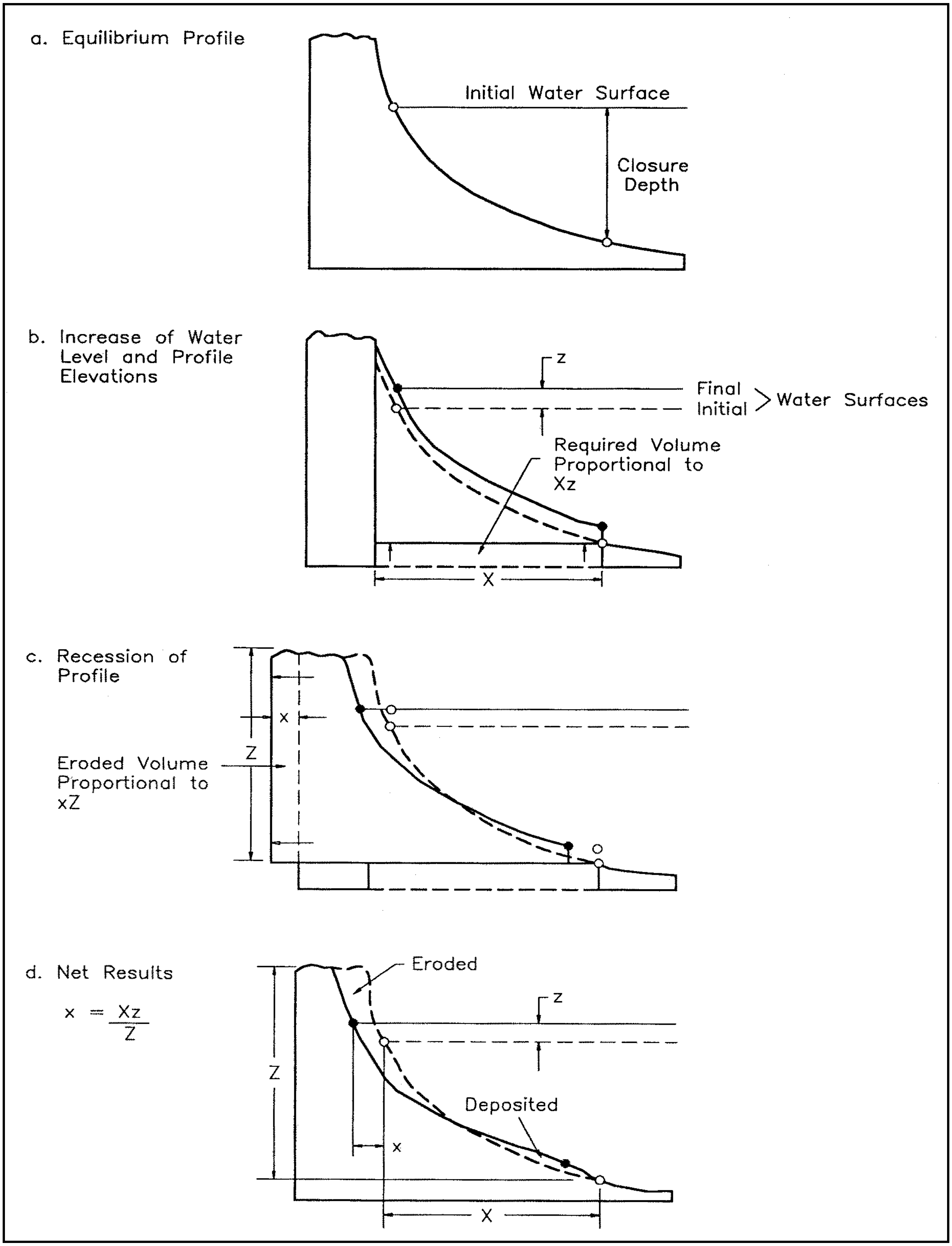

Figure IV-3-41. Profile adjustment in two stages, first vertical, then horizontal, demonstrating the basis for the Bruun Rule (Equation 3-6) (from Hands (1983)). Details discussed in the text

sediment is conserved in spite of the complex processes of local erosion versus deposition as bars migrate (Komar et al. 1991). Another strength is that it is a simple relationship, a geometric conclusion based only on water level. Despite its simplicity and numerous assumptions, it works remarkably well in many settings. Even with its shortcomings, it can be used to predict how beaches can respond to changes in sea level.

(4) Use of models to predict shoreline recession. Although field studies have confirmed the assumptions made by Bruun and others concerning translations of the shoreface, there has been no convincing demonstration that the models can predict shoreline recession rates. Komar et al. (1991) cite several reasons for the inability to use the models as predictive tools:

(a) Existence of a considerable time lag of the beach response following a sustained water level rise (as shown by Hands (1983) for Lake Michigan).

(b) Uncertainty in the selection of the parameters used in the equations (in particular, closure depth).

(c) Local complexities of sediment budget considerations in the sand budget.

(5) Recommendations. More field and laboratory studies are needed to better evaluate the response of beaches to rising (and falling) sea level. For example, it would be valuable to reoccupy the profile lines monitored by Hands (1976, 1979, 1980) in Lake Michigan in the 1970's to determine how the shores have responded to the high water of the mid-1980's and to the subsequent drop in the early 1990's. In addition, conceptual advances need to be incorporated in the theoretical models. How sediment has moved onshore in some locations following sea level rise also needs to be evaluated, because there is evidence that in some areas beach sand compositions reflect offshore rather than onshore sources (Komar et al. 1991).

g. Equilibrium profiles on sandy coasts.

(1) General characteristics and assumptions. The existence of an equilibrium shoreface profile (sometimes called equilibrium *beach* profile) is a basic assumption of many conceptual and numerical coastal models. Dean (1990) listed characteristic features of profiles:

(a) Profiles tend to be concave upwards.

(b) Fine sand is associated with mild slopes and coarse sand with steep slopes.

(c) The beach (above the surf zone) is approximately planar.

(d) Steep waves result in milder inshore slopes and a tendency for bar formation.

The main assumption underlying the concept of the shoreface equilibrium profile is that the seafloor is in equilibrium with *average* wave conditions. Presumably, the term *equilibrium* is meant to indicate a situation in which water level, waves, temperature, etc., are held constant for a sufficient time such that the beach profile arrives at a final, stable shape (Larson and Kraus 1989a). Larson (1991) described the profile as: “A beach of specific grain size, if exposed to constant forcing conditions, normally assumed to be short-period breaking waves, will develop a profile shape that displays no net change in time.” This concept ignores the fact that, in addition to wave action, many other processes affect sediment transport. These simplifications, however, may represent the real strength of the concept because it has proven to be a useful way to characterize the shape of the shoreface in many locations around the world.

(2) Shape. Based on studies of beaches in many environments, Bruun (1954) and Dean (1976, 1977) have shown that many ocean beach profiles exhibit a concave shape such that the depth varies as the two-thirds power of distance offshore along the submerged portions:

$$h = A y^{2/3} \qquad \text{(IV-3-7)}$$

where

h = water depth (m) at distance y (m) from the shoreline

A = a scale parameter that depends mainly on sediment characteristics

This surprisingly simple expression asserts, in effect, that beach profile shape can be calculated from sediment characteristics (particle size or fall velocity) alone. Moore (1982) graphically related the parameter A, sometimes called the *profile shape parameter,* to the median grain size d_{50}. Hanson and Kraus (1989) approximated Moore's curve by a series of lines grouped as a function of the median nearshore grain size d_{50} (in mm):

$$\begin{aligned} A &= 0.41 (d_{50})^{0.94} \quad , \quad d_{50} < 0.4 \\ A &= 0.23 (d_{50})^{0.32} \quad , \quad 0.4 \le d_{50} < 10.0 \\ A &= 0.23 (d_{50})^{0.28} \quad , \quad 10.0 \le d_{50} < 40.0 \\ A &= 0.46 (d_{50})^{0.11} \quad , \quad 40.0 \le d_{50} \end{aligned} \qquad \text{(IV-3-8)}$$

Note that A has unit dependence ($m^{1/3}$). Equation 3-8 is for SI (metric) values only. Different equations must be used for English units.

Table III-3-3 is a summary of recommended A values, and a more detailed discussion of methods to compute equilibrium profiles is provided in Part III-3.

(3) Discussion of assumptions. Pilkey et al. (1993), in a detailed examination of the concept of the equilibrium shoreface profile, contended that several assumptions must hold true for the concept to be valid:

(a) *Assumption 1: All sediment movement is driven by incoming wave orbitals acting on a sandy shoreface.*

This assumption is incorrect because research by Wright et al. (1991) showed that sediment movement on the shoreface is an exceedingly complex phenomenon, driven by a wide range of wave, tidal, and gravity currents. Even in locations where the wave orbitals are responsible for mobilizing the sand, bottom currents frequently determine where the sand will go.

(b) *Assumption 2: Existence of closure depth and no net cross-shore (i.e., shore-normal) transport of sediment to and from the shoreface.*

Pilkey et al. (1993) state that this assumption is also invalid because considerable field evidence has shown that large volumes of sand may frequently move beyond the closure depth. Such movement can occur during both fair weather and storm periods, although offshore-directed storm flows are most likely the prime transport agent. Pilkey et al. cite studies in the Gulf of Mexico that measured offshore bottom currents of up to 200 cm/sec and sediment transport to the edge of the continental shelf. The amount of sediment moved offshore was large, but it was spread over such a large area that the change in sea bed elevation could not be detected by standard profiling methods.[1] Wright, Xu, and Madsen (1994) measured significant across-shelf benthic transport on the inner shelf of the Middle Atlantic Bight during the Halloween storm of 1991.

(c) *Assumption 3: There exists a sand-rich shoreface; the underlying and offshore geology must not play a part in determining the shape of the profile.*

Possibly the most important of the assumptions implicit in the equilibrium profile concept is that the entire profile is sand-rich, without excessive areas of hard bottom or mud within the active profile. Clearly these conditions do not apply in many parts of the world. Coasts that have limited sand supplies, such as much of the U.S. Atlantic margin, are significantly influenced by the geologic framework occurring underneath and in front of the shoreface. Many of the east coast barriers are perched on a platform of ancient sediment. Depending upon the physical state, this underlying platform can act as a subaqueous headland or hardground that dictates the shape of the shoreface profile and controls beach dynamics and the composition of the sediment. Niederoda, Swift, and Hopkins (1985) believed that the seaward-thinning and fining veneer of modern shoreface sediments is ephemeral and is easily removed from the shoreface during major storms. During storms, Holocene and Pleistocene strata cropping out on the shoreface provide the immediate source of the bulk of barrier sands. Swift (1976) used the term *shoreface bypassing* to describe the process of older units supplying sediment to the shoreface of barrier islands. Pilkey et al. (1993) contend that:

> ...a detailed survey of the world's shorefaces would show that the sand rich shoreface required by the equilibrium profile model is an exception rather than the rule. Instead, most shorefaces are underlain by older, consolidated or semi-consolidated units covered by only a relatively thin veneer of modern shoreface sands. These older units are a primary control on the shape of the shoreface profile. The profile shape is not determined by simple wave interaction with the relatively thin sand cover. Rather, the shape of the shoreface in these sediment poor areas is determined by a complex interaction between underlying geology, modern sand cover, and highly variable (and often highly diffracted and refracted) incoming wave climate. (p. 271)

(d) *Assumption 4: If a shoreface is, in fact, sand-rich, the smoothed profile described by the equilibrium profile equation (ignoring bars and troughs) must provide a useful approximation of the real shoreface shape.*

In addressing this assumption, Pilkey et al. (1993) cited studies conducted on the Gold Coast, in Queensland, Australia. The Gold Coast shoreface is sand-rich to well beyond a depth of 30 m. Without being directly influenced by underlying geology, the shoreface is highly dynamic. As a consequence, the Gold Coast shoreface shape cannot be described by one equilibrium profile; rather, it is best described by an ever-changing regime profile. Pilkey et al. concluded:

> The local shoreface profile shapes are entirely controlled by relative wave energy "thresholds"; for the sediment properties have not changed at all. Thus principal changes to the shoreface profiles of the Gold Coast are driven by wave power history with some modification by currents, and not by sediment size, or its parameter *A*, as defined within the equilibrium profile concept. (p. 272).

[1] This latter statement underscores how important it is to develop improved methods to detect and measure sediment movement in deep water.

(4) General comments.

(a) The idea of a profile only adjusting to waves is fundamentally wrong as shown by Wright et al. (1991) and others. However, although the physical basis for the equilibrium profile concept is weak, critics of this approach have not proven that it always results in highly erroneous answers.

(b) Before the use of the equilibrium profile, coastal engineers had no way to predict beach change other than using crude approximations (e.g., sand loss of 1 cu yd/ft of beach retreat). The approximations were inadequate. Surveys from around the world have shown that shoreface profiles display a characteristic shape that differs with locality but is relatively stable for a particular place (i.e., Duck, North Carolina). With many caveats (which are usually stated, then ignored), a profile can be reasonably represented by the equilibrium equation. The fit between the profile and the real seafloor on a daily, seasonal, and storm variation basis may not be perfect, but the differences may not matter in the long term.

(c) One critical problem for coastal engineers is to predict what a sequence of waves (storm) will do to a locality when little is known about the particular shape of the pre-storm beach. For this reason, numerical models like SBEACH (Larson and Kraus 1989a), despite their reliance on the equilibrium profile concept, are still useful. The models allow a researcher to explore storm impact on a location using a general approximation of the beach. The method is very crude - however, the resulting numbers are of the right order of magnitude when compared with field data from many locations.

(d) Answers from the present models are not exact, and researchers still have much to learn about the weakness of the models and about physical processes responsible for the changes. Nevertheless, the models do work and they do provide numbers that are of the correct magnitudes when run by careful operators. Users of shoreface models must be aware of the limitations of the models and of special conditions that may exist at their project sites. In particular, profile-based numerical models are likely to be inadequate in locations where processes other than wave-orbital transport predominate.

h . Depth of closure.

(1) Background.

(a) *Depth of closure* is a concept that is often misinterpreted and misused. For engineering practice, depth of closure is commonly defined as the minimum water depth at which no measurable or significant change in bottom depth occurs (Stauble et al. 1993). The word *significant* in this definition is important because it leaves considerable room for interpretation. "Closure" has erroneously been interpreted to mean the depth at which no sediment moves on- or offshore, although numerous field studies have verified that much sediment moves in deep water (Wright et al. 1991). Another complication is introduced by the fact that it is impossible to define a single depth of closure for a project site because "closure" moves depending on waves and other hydrodynamic forces. Therefore, it is invalid to assume that "closure" is a single fixed depth at a project site or a stretch of coastline.

(b) Closure depth is used in a number of applications such as the placement of mounds of dredged material, beach fill, placement of ocean outfalls, and the calculation of sediment budgets.

(2) Energy factors. As discussed above, the primary assumption behind the concept of the shoreface equilibrium profile is that sediment movement and the resultant changes in bottom elevation are a function of wave properties and sediment grain size. Therefore, the active portion of the shoreface varies in width throughout the year depending on wave conditions. In effect, "closure" is a time-dependent quantity that may be predicted based on wave climatology or may be interpreted statistically using profile surveys.

(3) Time considerations. The energy-dependent nature of the active portion of the shoreface requires us to consider return period. The closure depth that accommodates the 100-year storm will be much deeper than one that merely needs to include the 10-year storm. Therefore, a closure depth must be chosen in light of a project's engineering requirements and design life. For example, if a berm is to be built in deep water where it will be immune from wave resuspension, what is the minimum depth at which it should be placed? This is an important question because of the high costs of transporting material and disposing of it at sea. It would be tempting to use a safe criterion such as the 100- or 500-year storm, but excessive costs may force the project engineer to consider a shallower site that may be stable only for shorter return period events.

(4) Predictive methods.

(a) Hallermeier (1977, 1978, 1981a, 1981b, 1981c), using laboratory tests and limited field data, introduced equations to predict the limits of extreme wave-related sediment movement. He calculated two limits, d_ℓ and d_i, that included a buffer region on the shoreface called the shoal zone. Landward of d_ℓ, significant alongshore transport and intense onshore-offshore sediment transport occur (the littoral zone). Within the shoal zone, expected waves have neither a strong nor a negligible effect on the sandy bed during a typical annual cycle of wave action. Seaward of d_i, only insignificant onshore-offshore transport by waves occurs. The deeper limit was based on the median nearshore storm wave height (and the associated wave period). The boundary between the shoal zone and the littoral zone (d_ℓ) as defined represents the annual depth of closure. Hallermeier (1978) suggested an analytical approximation, using linear wave theory for shoaling waves, to predict an *annual* value of d_ℓ:

$$d_\ell = 2.28 H_e - 68.5 \left(\frac{H_e^2}{g T_e^2}\right) \tag{IV-3-9}$$

where

d_ℓ = annual depth of closure below mean low water

H_e = non-breaking significant wave height that is exceeded 12 hr per year (0.137 % of the time)

T_e = associated wave period

g = acceleration due to gravity

According to Equation 3-9, d_ℓ is primarily dependent on wave height with an adjustment for wave steepness. Hallermeier (1978) proposed using the 12-hr exceeded wave height, which allowed sufficient duration for "moderate adjustment towards profile equilibrium." Equation 3-9 is based on quartz sand with a submerged density of γ' = 1.6 and a median diameter between 0.16 and 0.42 mm, which typifies conditions in the nearshore for many beaches. If the grain size is larger than 0.42 mm, Equation 3-9 may not be appropriate. Because d_ℓ was derived from linear wave theory for shoaling waves, d_ℓ must be seaward of the influence of intense wave-induced nearshore circulation. However, because of various factors, Hallermeier (1978) "proposed that the calculated d_ℓ be used as a minimum estimate of profile close-out depth with respect to low(er) tide level." Because tidal or wind-induced currents may increase wave-induced near-bed flow velocities, Hallermeier suggested using mean low water (mlw) as a reference water level to obtain a conservative depth of closure. Note that Hallermeier's equations critically depend on the quality of wave data at a site. The reader is cautioned that Hallermeier's equations can be expressed in various forms depending on the assumptions made, the datums used as reference levels, and available wave data. The reader is referred to his original papers for clarification and for details of his assumptions. The equations may not be applicable at sites where currents are more important at moving sand than wave-induced flows.

(b) At the Lake Michigan sites that Hands (1983) surveyed, the closure depth was equal to about twice the height of the 5-year return period wave height (H_5):

$$Z \simeq 2H_5 \qquad \text{(IV-3-10)}$$

In the absence of strong empirical evidence as to the correct closure depth, this relationship is recommended as a rule of thumb to estimate the 5-year profile response under Great Lakes conditions. The return period of the wave height should approximate the design life of interest. For example, the 20-year closure depth would be estimated by doubling the 20-year return period wave height ($Z \simeq 2H_{20}$).

(5) Empirical determination.

(a) When cross-shore surveys covering several years are available for a project site, closure is best determined by plotting and analyzing the profiles. The closure depth computed in this manner reflects the influence of storms as well as of calmer conditions. Kraus and Harikai (1983) evaluated the depth of closure as the minimum depth where the standard deviation in depth change decreased markedly to a near-constant value. Using this procedure, they interpreted the landward region where the standard deviation increased to be the active profile where the seafloor was influenced by gravity waves and storm-driven water level changes. The offshore region of smaller and nearly constant standard deviation was primarily influenced by lower frequency sediment-transporting processes such as shelf and oceanic currents (Stauble et al. 1993). It must be noted that the smaller standard deviation values fall within the limit of measurement accuracy. This suggests that it is not possible to specify a closure depth unambiguously because of operational limits of offshore profiling hardware and procedures.

(b) An example of how closure was determined empirically at Ocean City, Maryland, is shown in Figure IV-3-42 (from Stauble et al. (1993)). A clear reduction in standard deviation occurs at a depth of about 5.5 - 6 m. Above the ~5.5 m depth, the profile exhibits large variability, indicating active wave erosion, deposition, and littoral transport. Deeper (and seaward) of this zone, the lower and relatively constant deviation of about 7 - 10 cm is within the measurement error of the sled surveys. Nevertheless, despite the inability to precisely measure seafloor changes in this offshore region, it is apparent that less energetic erosion and sedimentation take place here than in water shallower than ~5.5 m. This does not mean that there is no sediment transport in deep water, just that the sled surveys are unable to measure it. For the 5.6 km of shore surveyed at Ocean City, the depth of closure ranged between 5.5 and 7.5 m. Scatter plots indicated that the average closure depth was 6 m.

(c) Presumably, conducting surveys over a longer time span at Ocean City would reveal seafloor changes deeper than ~6 m, depending on storms that passed the region. However, Stauble et al. (1993) noted that the "Halloween Storm" of October 29 to November 2, 1991 generated waves of peak period (T_p) 19.7 sec, extraordinarily long compared to normal conditions along the central Atlantic coast. Therefore, the profiles may already reflect the effects of an unusually severe storm.

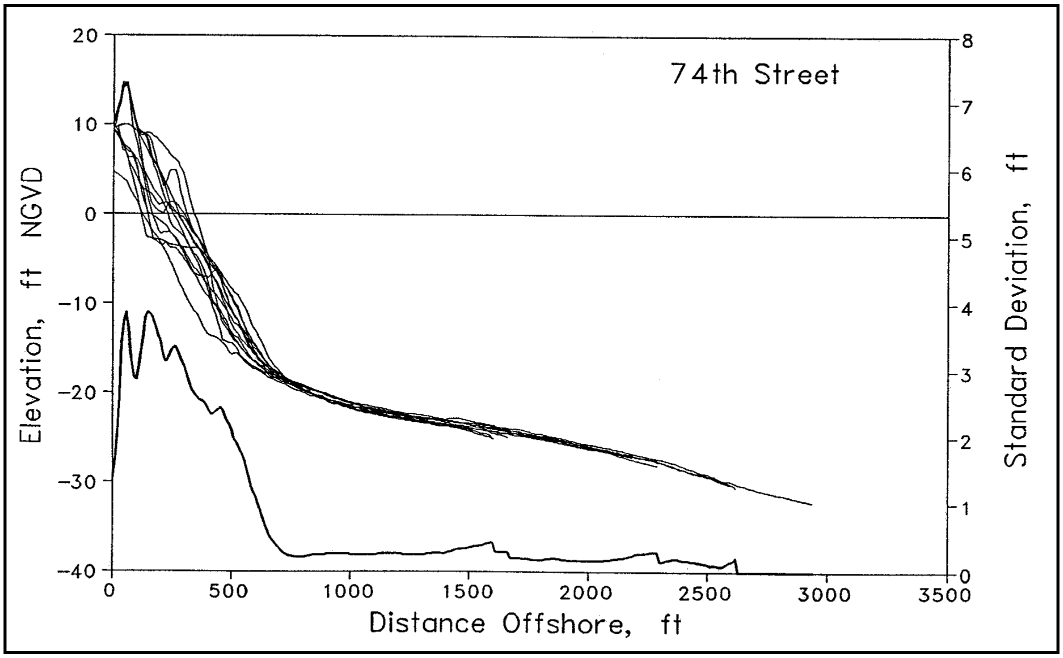

Figure IV-3-42. Profile surveys and standard deviation of seafloor elevation at 74th Street, Ocean City, Maryland (from Stauble et al. (1993)). Surveys conducted from 1988 to 1992. Large changes above the datum were caused by beach fill placement and storm erosion. Figure discussed in the text

(d) Figure IV-3-43 is an example of profiles from St. Joseph, Michigan, on the east shore of Lake Michigan. Along Line 14, dramatic bar movement occurs as far as 760 m offshore to a depth of 7.6 m with respect to International Great Lakes Datum (IGLD) 1985. This is where an abrupt decrease in standard deviation of lake floor elevation occurs and can be interpreted as closure depth. In September 1992, the mean water surface was 0.51 m above IGLD 85. Therefore, closure was around 7.9 - 8.2 m below *water* level.

(e) In the Great Lakes, water levels fluctuate over multi-year cycles. This raises some fundamental difficulties in calculating closure based on profile surveys. Presumably, during a period of high lake level, the zone of active sand movement would be higher on the shoreface than during a time of low lake level (this assumes similar wave conditions). Therefore, the depth where superimposed profiles converge should reflect the *deepest* limit of active shoreface sand movement. This would be a conservative value, but *only with respect to the hydrologic conditions that occurred during the survey program*. Presumably, if lake level dropped further at a later date, sediment movement might occur deeper on the shoreface. This suggests that closure on the lakes should be chosen to reflect the *lowest* likely water level that is expected to occur during the life of a project. (Note that this consideration does not arise on ocean coasts because year-to-year changes in relative sea level are minor, well within the error bounds of sled surveys. Sea level does change throughout the year because of thermal expansion, freshwater runoff, and other factors as discussed in Part IV-1, but the multi-year mean is essentially stable.) In summary, determining closure depth in the Great Lakes is problematic because of changing water levels, and more research is needed to develop procedures that accommodate these non-periodic lake level fluctuations.

(f) The variation of closure depth at approximately 100 profile lines along the south shore of Long Island is plotted in Figure IV-3-44. Generally the depth increases towards the east, with Rockaway Beach averaging 5.0 m below NGVD and the Montauk zone averaging 7.6 m. These values were based on

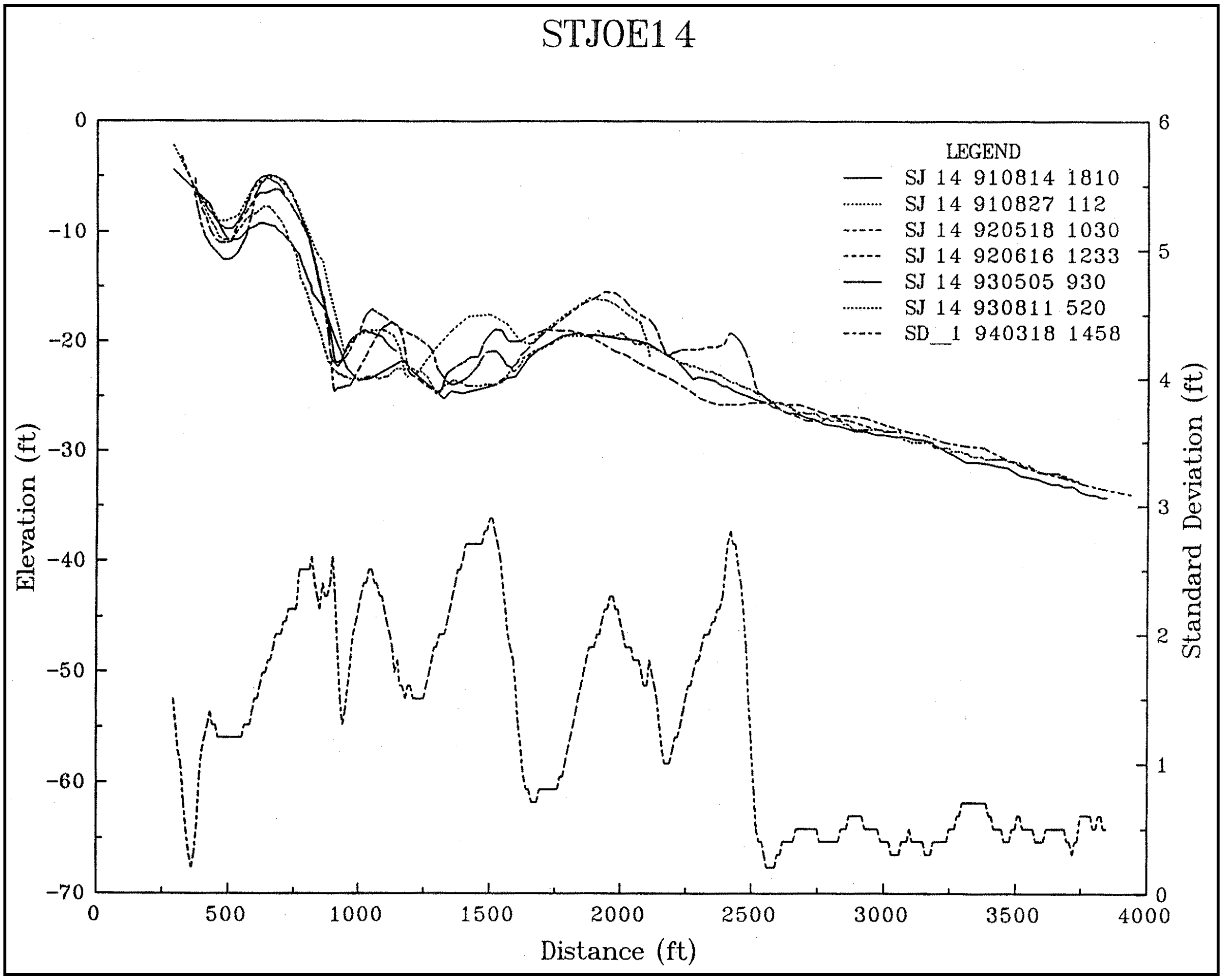

Figure IV-3-43. Profile surveys and standard deviation of lake floor elevation at St. Joseph, Michigan, on the east shore of Lake Michigan. Profiles are referenced to International Great Lakes Datum (IGLD) 1985. Surveys conducted between 1991 and 1994 (Nairn et al. 1997). Figure discussed in the text

surveys in 1995 and 1996 (Morang 1998). The depth increase toward the east is expected because wave energy is greater there than at the west end of Long Island.

i. Longshore sediment movement.

The reader is referred to Part III-2 and to *Coastal Sediment Transport* (EM 1110-2-1502) for a detailed treatment of longshore transport.

j. Summary.

(1) A model of shoreface morphodynamics for micro- and low-mesotidal sandy coasts has been developed by Wright and Short (1984). The six stages of the model (Figure IV-3-34) illustrate the response of sandy beaches to various wave conditions.

(2) Sediment movement on the shoreface is a very complicated phenomenon. It is a result of numerous hydrodynamic processes, among which are: (a) wave orbital interactions with bottom sediments and with

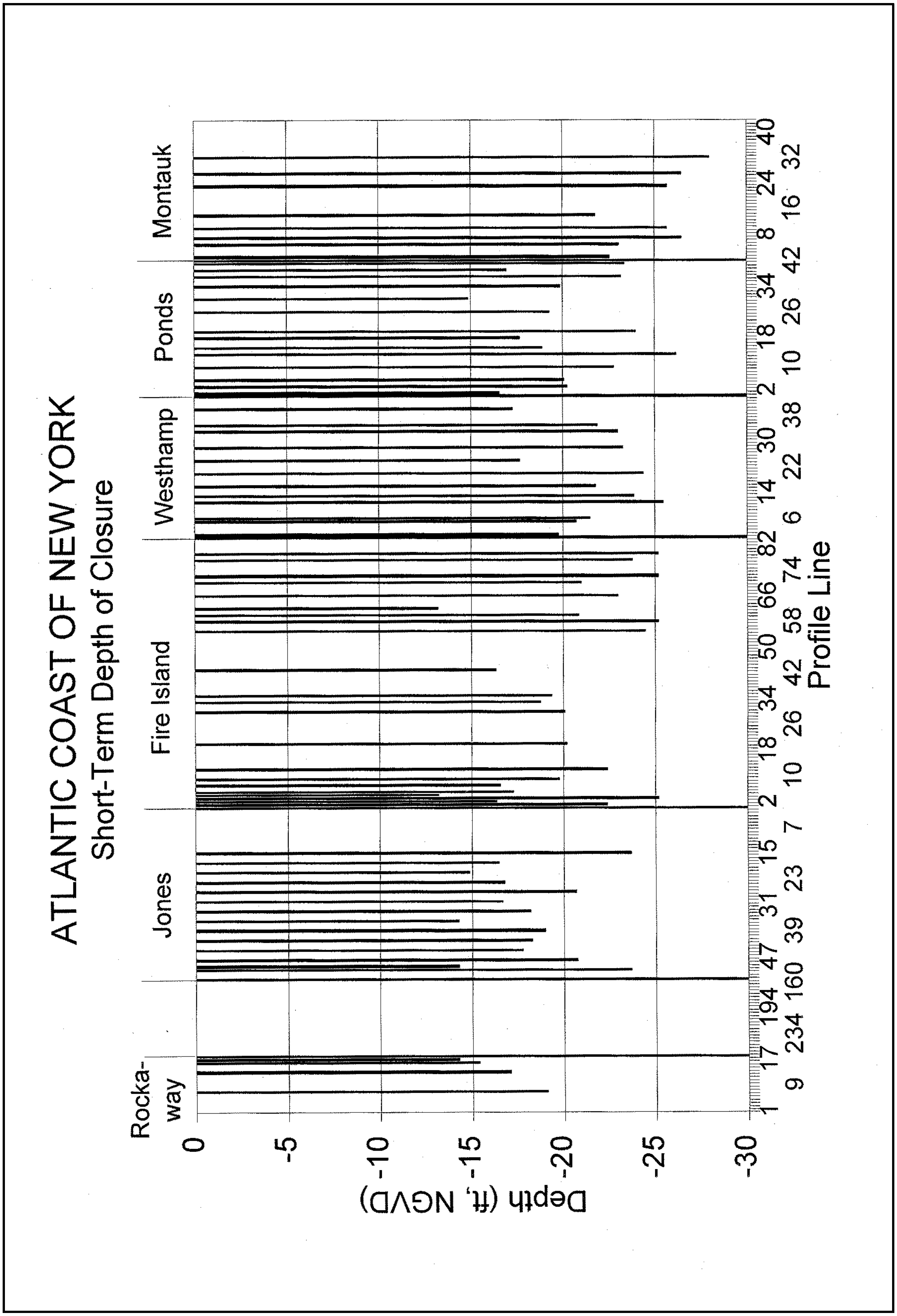

Figure IV-3-44. Variation in short-term closure depth along the south shore of Long Island, New York, computed for four survey dates in 1995 and 1996

wave-induced longshore currents; (b) wind-induced longshore currents; (c) rip currents; (d) tidal currents; (e) storm surge ebb currents; (f) gravity-driven currents; (g) wind-induced upwelling and downwelling; (h) wave-induced upwelling and downwelling; and (i) gravity-induced downslope transport.

(3) The Bruun Rule (Equation 3-5 or 3-6) is a model of shoreface response to rising sea level. Despite the model's simplicity, it helps explain how barriers have accommodated rising sea level by translating upward and landward. A limitation is that the model does not address *when* the predicted shore response will occur (Hands 1983). It merely reveals the horizontal distance the shoreline must *ultimately* move to reestablish the equilibrium profile at its new elevation under the stated assumptions.

(4) The concept of the equilibrium shoreface profile applies to sandy coasts primarily shaped by wave action. It can be expressed by a simple equation (Equation 3-7) which depends only on sediment characteristics. Although the physical basis for the equilibrium profile concept is weak, it is a powerful tool because models based on the concept produce resulting numbers that are of the right order of magnitude when compared with field data from many locations.

(5) *Closure* is a concept that is often misinterpreted and misused. For engineering practice, depth of closure is commonly defined as the minimum water depth at which no measurable or significant change in bottom depth occurs (Stauble et al. 1993). Closure can be computed by two methods: (a) analytical approximations such as those developed by Hallermeier (1978), which are based on wave statistics at a project site (Equation 3-10); or (b) empirical methods based on cross-shore survey profile data. When profiles are superimposed, a minimum value for closure can be interpreted as the depth where the standard deviation in depth change decreases markedly to a near-constant value. Both methods have weaknesses. Hallermeier's analytical equations depend on the quality of wave data. Empirical determinations depend on the availability of several years of profile data at a site. Determining closure in the Great Lakes is problematic because lake levels fluctuate due to changing hydrographic conditions.

IV-3-6. References

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IV-3-7. Definition of Symbols

β	Gradient of the beach and surf zone
ε	Surf-scaling parameter (Equation IV-3-4) [dimensionless]
ρ_f	Mass density of fresh water (= 1,000kg/m^3 or 1.94 slugs/ft^3) [force-time^2/length^4]
ρ_s	Mass density of salt water (= 1,025 kg/m^3 or 2.0 slugs/ft^3) [force-time^2/length^4]
ω	Wave angular or radian frequency (= $2\pi/T$) [time^{-1}]
Ω	Modal state of the beach (Equation IV-3-3) [dimensionless]
A	Sediment scale or equilibrium profile parameter or profile shape parameter (Table III-3-3) [length$^{1/3}$]
a_b	Breaker amplitude [length]
B	Berm height of the eroded area [length]
d_ℓ	Annual depth of closure below mean low water (Equation IV-3-9) [length]
F'	Froude number
g	Gravitational acceleration (32.17 ft/sec^2, 9.807m/sec^2) [length/time^2]
h	Equilibrium beach profile depth (Equation IV-3-7) [length]
H_*	Water depth [length]
H_b	Wave height at breaking [length]
H_e	Non-breaking significant wave height that is exceeded 12 hr per year [length]
H_x	Wave height of the x-year return period [time]
h'	Depth of density interface [length]
L_*	Cross-shore distance to the water depth H_* [length]
R	Shoreline retreat (Equation IV-3-5) [length]
S	Increase in sea level [length]
T	Wave period [time]
T_e	Wave period associated with H_e [time]
U	Mean outflow velocity of upper layer (in case of stratified flow) [length/time]
$\overline{w_s}$	Sediment fall velocity [length/time]
x	Retreat of the profile, Bruun Rule (Equation IV-3-6) [length]
X	Horizontal distance of responding profile {Equation IV-3-6 and Figure IV-3-40) [length]

y	Equilibrium beach profile distance offshore (Equation IV-3-7) [length]
z	Change in water level [length]
Z	Closure depth (Equation IV-3-10) [length]
Z	Vertical distance of responding profile {Equation IV-3-6 and Figure IV-3-40) [length]

IV-3-8. Acknowledgments

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